3.2 **DEFINITIONS**, cont.

Revise or add the following definitions::

Permanent Loads – Loads and forces that are, or are assumed to be, either constant or varying over a long time interval upon completion of construction.

Transient Loads – Loads and forces that are, or are assumed to be, varying over a short time interval ~~or that redistribute under ultimate load~~.

3.3 NOTATION

3.3.1 General

Revise or add the following notations:

$T_{BaseConstr}$ = base construction temperature

3.3.2 Load and Load Designation

Revise as follows:

The following permanent and transient loads and forces shall be considered:

- Permanent Loads

CR = force effect due to creep
DD = downdrag force
DW = dead load of wearing surfaces and utilities
EH = horizontal earth pressure load
EL = miscellaneous locked-in force effects resulting from the construction process.
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill
PS = secondary forces from post-tensioning
SH = force effect due to shrinkage

- Transient Loads

BR = vehicular braking force
CE = vehicular centrifugal force
~~*CR* = creep~~
CT = vehicular collision force
CV = vessel collision force
EQ = earthquake load
FR = friction force
IC = ice load
IM = vehicular dynamic load allowance
LL = vehicular live load
LS = live load surcharge
PL = pedestrian live load
SE = force effect due to settlement
~~*SH* = shrinkage~~
TG = force effect due to temperature gradient
TU = force effect due to uniform temperature
WA = water load and stream pressure
WL = wind on live load
WS = wind load on structure

3.4.1 Load Factors and Load Combinations

Revise the 1st paragraph in Article 3.4.1 as follows:

- STRENGTH I—Basic load combination relating to the normal vehicular use of the bridge without wind

Revise the 2nd paragraph as follows:

- STRENGTH II—Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.
  a) <u>Distribution Factor—Load (DF) combination applies for superstructure design with load distribution factor tables in Articles 4.6.2.2, only.</u>
  b) <u>Lever Rule (LV), Substructure—Load (SUB) combination used for superstructure design when the lever rule is called for by the tables in Article 4.6.2.2, for substructure design, or whenever a whole number of traffic lanes are to be used.</u> <u>Live loads shall be placed in **a maximum of two** separate lanes chosen to create the most severe conditions.</u>

Revise as follows:

~~A reduced value of 0.50, applicable to all strength load combinations, specified for *TU*, *CR*, and *SH*, used when calculating force effects other than displacements at the strength limit state, represents an expected reduction of these force effects in conjunction with the inelastic response of the structure. The calculation of displacements for these loads utilizes a factor greater than 1.0 to avoid undersized joints, and bearings. The effect and significance of the temperature gradient remains unclear at this writing. Consult Article C3.12.3 for further information.~~

~~The permit vehicle should not be assumed to be the only vehicle on the bridge unless so assured by traffic control. See Article 4.6.2.2.4 regarding other traffic on the bridge simultaneously~~. <u>The vehicular braking force shall not be included in this load combination</u>.

a) Distribution Factor—<u>Multiple presence is already considered in the load distribution factor tables in Articles 4.6.2.2</u>.
b) <u>Lever Rule, Substructure —Multiple presence factors from Article 3.6.1.1.2 apply</u>.

Revise the 2nd paragraph, EXTREME EVENT, as follows:

- EXTREME EVENT I—Load combination including earthquake.

Revise as follows:

Although this limit state includes water loads, WA, the effects due to WA are considerably less significant than the effects on the structure stability due to degradation. Therefore, unless specific site conditions dictate otherwise, local pier scour and contraction scour depths should not be included in <u>the structural or geotechnical</u> design. However the effects due to degradation of the channel should be considered. Live load coincident with an earthquake is discussed elsewhere in this Article.

- EXTREME EVENT II—Load combination relating to ice load, collision by vessels and vehicles, and certain hydraulic events with a reduced live load other than that which is part of the vehicular collision load, *CT*.

The joint probability of these events is extremely low, and, therefore, the events are specified to be applied separately. Under these extreme conditions, the structure is expected to undergo considerable inelastic deformation by which locked-in-force effects due to TU, TG, ~~CR, SH~~ and SE are expected to be relieved. <u>The effects due to degradation scour, only, should be considered for both structural and geotechnical design.</u>

Revise the 2nd paragraph as follows:

FATIGUE I -Fatigue and fracture load combination relating to finite fatigue life and infinite fatigue life due to repetitive gravitational vehicular HL-93 truck live load and dynamic response under a single design truck having the axle spacing specified in Article 3.6.1.4.1. The load factors of 0.875 and 1.75 shall be used for finite fatigue life and infinite fatigue life, respectively.

Revise as follows:

The load factor applied to a single design truck, reflects a load level found to be representative of the truck population with respect to a large number of return cycles of stresses and to their cumulative effects in ~~steel~~ elements, components, and connections.

Infinite fatigue life is the design concept used for higher traffic volume bridges. The maximum fatigue stress range is kept lower than the constant-amplitude fatigue threshold to provide a theoretically infinite fatigue life. Finite fatigue life is the design concept used for lower traffic volume bridges. The effective fatigue stress range is kept lower than the fatigue resistance, which is a function of cycles and details, to provide a finite fatigue life.

A comprehensive comparison study of fatigue load moments for steel girder bridges using the *AASHTO LRFD Bridge Design Specifications* (3rd Edition, 2004) compared to the *AASHTO Standard Specifications* (17th Edition, 2002) was performed. From this parametric study, it is observed that the **LRFD** fatigue moments in an interior girder are about **60%** and **20%** less than that of the **Standard,** for finite fatigue life and infinite fatigue life, respectively.

To reflect past Caltrans' infinite fatigue life design practice using the *AASHTO Standard Specifications*, the load factor of 0.875 together with a revised Fatigue Resistance Equation (6.6.1.2.5-1a), and the load factor of 1.75 together with a revised Fatigue Resistance Equation (6.6.1.2.5-1b) should be used for infinite fatigue life and finite fatigue life in Fatigue I Limit State, respectively, for steel design. Those factors are based on the assumption that the maximum stress range is twice the live load stress range due to the passage of the fatigue truck specified in Article 3.6.1.2.2 with a constant spacing of 30.0 ft. between the 32.0-kips axles and derived by calibrating the LFRD fatigue design procedure to Caltrans past LFD design procedure. Figure C1 shows comparisons of fatigue resistance vs. number of cycles for a steel detail.

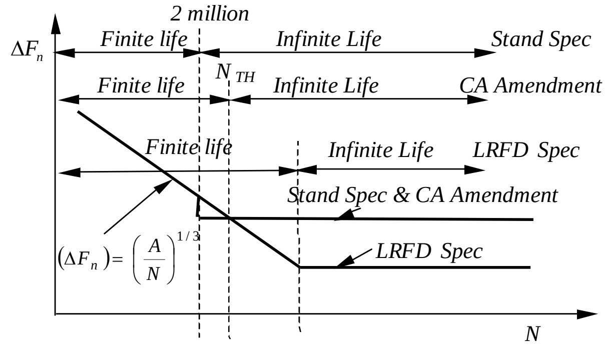

Figure C3.4.1-1 Fatigue Resistance vs. Number of Cycles.

(**3.4.1**, cont.)
FATIGUE II -fatigue and fracture load combination relating finite fatigue life due to repetitive gravitational vehicular P-9 truck live load and dynamic response under a single design truck having the axle spacing specified in Article 3.6.1.4.1.

(**C3.4.1**, cont.)
The load factor of 1.0 applied to a single design truck, reflects a load level found to be representative of the permit truck population with respect to a large number of return cycles of stresses and to their cumulative effects in elements, components, and connections.

Revise the 6th paragraph of Article 3.4.1 as follows:

The larger of the values provided for load factors of *TU*, ~~*CR* and *SH*~~ shall be used for deformations and the smaller values for all other effects. For simplified analysis of substructures in the strength limit state, a value of 0.50 for γ_{TU} may be used when calculating force effects, but shall be taken in conjunction with the gross moment of inertia in the columns or piers. When a refined analysis is completed for substructures in the strength limit state, a value of 1.0 for γ_{TU} shall be used in conjunction with a partially cracked moment of inertia determined by analysis. For substructures in the strength limit state, the value of 0.50 for γ_{PS}, γ_{CR}, and γ_{SH} may similarly be used when calculating force effects in non-segmental structures, but shall be taken in conjunction with the gross moment of inertia in the columns or piers.

Add two new paragraphs to the Commentary as follows:

PS, *CR*, *SH*, *TU* and *TG* are superimposed deformations as defined in Article 3.12. Load factors for *TU* and *TG* are as shown in Table 1. Load factors for PS, CR, and SH, are as shown in Table 2. For prestressed members in typical bridge types, secondary prestressing, creep and shrinkage are generally designed for in the service limit state. In segmental structures, *CR* and *SH* are factored by γ_P for *DC* because some analytic methods for time-dependent effects in segmental bridges are nonlinear.

The calculation of displacements for *TU* utilizes a factor greater than 1.0 to avoid undersizing joints, expansion devices, and bearings.

Revise Eq. 3.4.1-2 as follows:
DC+*DW*+*EH*+*EV*+*ES*+*WA*+*CR*+*SH*+*TG*+EL+PS

Revise Table 3.4.1-1 as follows:

Table 3.4.1-1 – Load Combinations and Load Factors

Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* ***PS*** ***CR*** ***SH***	LL_{HL93} *IM* *CE* *BR* *PL* *LS*	LL_{Permit} *IM* *CE*	*WA*	*WS*	*WL*	*FR*	*TU* ~~*CR*~~ ~~*SH*~~	*TG*	*SE*	*EQ* *IC* *CT* *CV* (use only one)
STRENGTH I	γ_p	1.75	**0.0**	1.0	**0.0**	**0.0**	1.0	0.50/ 1.20	γ_{TG}	γ_{SE}	**0.0**
STRENGTH II- DF, LVR,SUB	γ_p	**0.0**	1.35	1.0	**0.0**	**0.0**	1.0	0.50/ 1.20	γ_{TG}	γ_{SE}	**0.0**
STRENGTH III	γ_p	**0.0**	**0.0**	1.0	1.4	**0.0**	1.0	0.50/ 1.20	γ_{TG}	γ_{SE}	**0.0**
STRENGTH IV	γ_p	**0.0**	**0.0**	1.0	**0.0**	**0.0**	1.0	0.50/ 1.20	***0.0***	***0.0***	**0.0**
STRENGTH V	γ_p	1.35	**0.0**	1.0	0.4	1.0	1.0	0.50/ 1.20	γ_{TG}	γ_{SE}	**0.0**
EXTREME EVENT I	~~γ_p~~ **1.0**	~~γ_{EQ}~~ **0.0**	**0.0**	1.0	**0.0**	**0.0**	1.0	**0.0**	**0.0**	**0.0**	1.00 (EQ)
EXTREME EVENT II	~~γ_p~~ **1.0**	0.5	**0.0**	1.0	**0.0**	**0.0**	1.0	**0.0**	**0.0**	**0.0**	1.00 (IC or CT or CV)
SERVICE I	1.00	1.00	**0.00**	1.00	0.30	1.0	1.0	1.00/ 1.20	γ_{TG}	γ_{SE}	**0.0**
SERVICE II	1.00	1.30	**0.00**	1.00	**0.0**	**0.0**	1.0	1.00/ 1.20	**0.0**	**0.0**	**0.0**
SERVICE III	1.00	0.80	**0.00**	1.00	**0.0**	**0.0**	1.0	1.00/ 1.20	γ_{TG}	γ_{SE}	**0.0**
SERVICE IV	1.00	**0.00**	**0.00**	1.00	0.70	**0.0**	1.0	1.00/ 1.20	**0.0**	1.0	**0.0**
FATIGUE I— LL_{HL93}, *IM* & *CE* ONLY	**0.00**	~~0.75~~ 0.875/ 1.75	**0.00**	**0.00**	**0.00**	**0.0**	**0.0**	**0.00**	**0.0**	**0.0**	**0.00**
FATIGUE II— LL_{Permit}, & *IM*	**0.00**	**0.00**	**1.00**	**0.00**	**0.00**	**0.0**	**0.0**	**0.00**	**0.0**	**0.0**	**0.00**

Revise Table 3.4.1-2 as follows:

Table 3.4.1-2 Load Factors for Permanent Loads, γ_p

Type of Load		Load Factor Maximum	Load Factor Minimum
DC: Component and Attachments ***DC*: Strength IV, only**		1.25 1.50	0.90 0.90
DD: Downdrag	**Piles, α Tomlison Method** **Piles, λ Method** **Drilled Shafts, O'Neill and Reese (*1999*) Method**	1.40 1.05 1.25	0.25 0.30 0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure • Active • At-Rest		 1.50 1.35	 0.90 0.90
EL: Locked-in ~~Erection~~ <u>Construction</u> Stresses		1.00	1.00
EV: Vertical Earth Pressure • Overall Stability • Retaining Walls and Abutments • Rigid Buried Structure • Rigid Frame • Flexible Buried Structures other than Metal Box Culverts • Flexible Metal Box Culverts		 1.00 1.35 1.30 1.35 1.95 1.50	 N/A 1.00 0.90 0.90 0.90 0.90
ES: Earth Surcharge		1.50	0.75
<u>*PS*: Secondary Forces from Post-tensioning</u> • <u>Substructure-Supporting Non-segmental Superstructures When Using I_g</u> • <u>All Other Structures</u>		 <u>0.50</u> <u>1.00</u>	 <u>0.50</u> <u>1.00</u>
<u>*CR*: Force due to Creep</u> *SH*: Force due to Shrinkage • <u>Superstructures - Segmental</u> • <u>Superstructures-Non-segmental</u> • <u>Substructures-Supporting Segmental Superstructures (see 3.12.4, 3.12.5)</u> • <u>Substructure-Supporting Non-segmental Superstructures When Using I_g</u> <u>*CR*, *SH*: Strength IV Only</u>		 <u>1.25</u> <u>1.00</u> <u>1.25</u> <u>0.50</u> <u>1.50</u>	 <u>0.90</u> <u>1.00</u> <u>0.90</u> <u>0.50</u> <u>0.90</u>

3.4.1 (cont.)

Delete the last paragraph as follows:

~~The load factor for live load in Extreme Event Load Combination I, γ_{EQ}, shall be determined on a project-by-project basis"~~

3.6.1.1.2 Multiple Presence of Live Load

Revise the 3rd paragraph as follows:

The factors specified in Table 1 shall not be applied in conjunction with approximate load distribution factors specified in Articles 4.6.2.2 and 4.6.2.3, except where the lever rule is used or where special requirements for exterior beams in beam-slab bridges, specified in Article 4.6.2.2.2d, are used. <u>Furthermore, the factors specified in Table 1 shall not be applied to the design of culvert top slabs when using the equivalent strip method as specified in Article 4.6.2.1.</u>

C3.6.1.1.2

Add a new last paragraph as follows:

<u>Reinforced Box Culverts are designed on a unit-width basis. Each unit-width must be capable of withstanding the applied truck load regardless of how many adjacent lanes are loaded. Furthermore, live load forces overlap but dissipate through the fill, and generally are less significant than loads due to fill.</u>

3.6.1.2.6 Distribution of Wheel Loads Through Earth Fills

Revise as follows:

Where the depth of fill is less than 2.0 ft., live loads shall be distributed to the top slabs of culverts as specified in Article 4.6.2.10.

In lieu of a more precise analysis, or the use of other acceptable approximate methods of load distribution permitted in Section 12, where the depth of fill is 2.0 ft. or greater, wheel loads may be considered to be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area, as specified in Article 3.6.1.2.5, and increased by ~~either 1.15 times the depth of the fill in select granular backfill, or~~ the depth of the fill in all other cases. The provisions of Article~~s 3.6.1.1.2 and~~ 3.6.1.3 shall apply.

C3.6.1.2.6

Add a new 3rd and 4th paragraphs as follows:

Select granular backfill is not used for embankments in California.

The multiple presence factor should not be applied when designing the top slab of culverts.

3.6.1.3 Application of Design Vehicular Live Loads

3.6.1.3.1 General

Add a new 4th bullet as follows:

- For both negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only, 100 percent of the effect of two design tandems spaced anywhere from 26.0 ft. to 40 ft. from the lead axle of one tandem to the rear axle of the other, combined with the design lane load specified in Article 3.6.1.2.4.

C3.6.1.3.1

Revise the 3rd paragraph as follows:

The notional design loads were based on the information described in Article C3.6.1.2.1, which contained data on "low boy" type vehicles weighing up to about 110 kip. ~~Where multiple lanes of heavier versions of this type of vehicle are considered probable, consideration should be given to investigating negative moment and reactions at interior supports for pairs of the design tandem spaced from 26.0 ft. to 40.0 ft. apart, combined with the design lane load specified in Article 3.6.1.2.4. One hundred percent of the combined effect of the design tandems and the design lane load should be used.~~ In California, side-by-side occurrences of the "low boy" truck configuration are routinely found. This amendment is consistent with Article 3.6.1.2.1, will control negative bending serviceability in two-span continuous structures with 20-ft to 60-ft span lengths, and should not be considered a replacement for the Strength II Load Combination.

3.6.1.3.3 *Design Loads for Decks, Deck Systems, and the Top Slabs of Box Culverts*

C3.6.1.3.3

Add a new 5th paragraph as follows:

The force effects due to one 32-k axle on the strip-widths specified in Table 4.6.2.1.3-1, were found to be similar to Caltrans' past practice and envelop two 24-k axles 4-0 o.c. (design tandem). Also, the 54-k tandem axle of the permit vehicle typically doesn't control deck designs when applying the appropriate load factors or allowable stresses.

3.6.1.3.4 Deck Overhang Load

Delete the 1st paragraph as follows:

~~For the design of deck overhangs with a cantilever, not exceeding 6.0 ft. from the centerline of the exterior girder to the face of a structurally continuous concrete railing, the outside row of wheel loads may be replaced with a uniformly distributed line load of 1.0 klf intensity, located 1.0 ft. from the face of the railing.~~

C3.6.1.3.4

Add a new last paragraph as follows:

Barriers shall not be considered as continuous structural elements.

3.6.1.4 Fatigue Load

3.6.1.4.1 Magnitude and Configuration

Revise the 1st paragraph as follows:

For the Fatigue I limit state, t~~The~~ fatigue load shall be one design truck or axles thereof specified in Article 3.6.1.2.2, but with a constant spacing of 30.0 ft. between the 32.0-kip axles.

For the Fatigue II limit state, the fatigue load shall be one Permit truck as specified in Figure 1.

C3.6.1.4.1

Add the following paragraph:

The fatigue Permit Truck specified in Figure 1 represents the majority of permit trucks allowed in California.

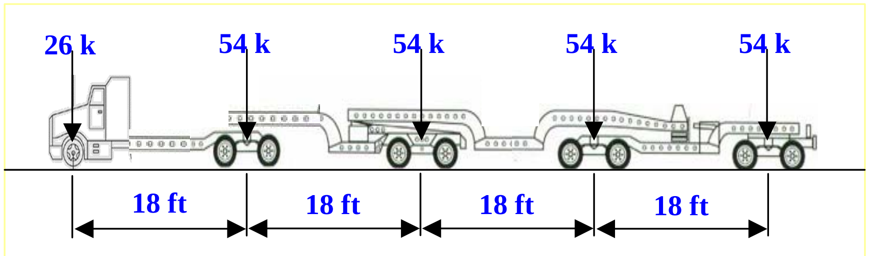

Figure 3.6.1.4.1-1. Fatigue Permit Truck.

3.6.1.4.2 Frequency

Add the following as the last paragraph:

In the absence of specific data, ***ADTT*** should be taken as 2500 and 20, for the Fatigue I limit state and the Fatigue II limit state, respectively.

C3.6.1.4.2

Add the following as the last paragraph:

ADTT of 2500 for the HS-20 fatigue truck has been successfully used for design of new structures and widenings in California. Based on the variation of sizes, weight and volumes of P5 through P13 Permit trucks operating in California, along with the growth rate of 1% within the 75-year design life; the volumes of P5 through P13 trucks are conservatively converted to an equivalent fatigue P9 permit truck with an *ADTT* = 20.

3.6.1.6 Pedestrian Loads

Add a new 4th paragraph as follows:

The frequency of pedestrian footfall loads in either the vertical or transverse lateral direction shall not resonate with the natural frequencies of the structure.

C3.6.1.6

Revise the 1st paragraph as follows:

See the provisions of Article 3.6.1.1.2 for applying the pedestrian loads in combination with the vehicular live load. The pedestrian load need not be used in the Strength II load combination.

Add a new 4th paragraph as follows:

Footfall has been estimated to have a frequency of 2 Hz in the vertical direction, and 0.67 Hz in the transverse lateral direction. Therefore, the fundamental frequency of the structure should be a minimum of 3 HZ and 1.3 Hz in the vertical and lateral directions respectively, unless detailed analysis justifies otherwise.

Add a new Article as follows:

3.6.1.8 Permit Vehicles

3.6.1.8.1 General

C3.6.1.8

Permit design live loads, or P loads, are special design vehicular loads. The weights and spacings of axles and wheels for the overload truck shall be as specified in Figure 3.6.1.8.1-1.

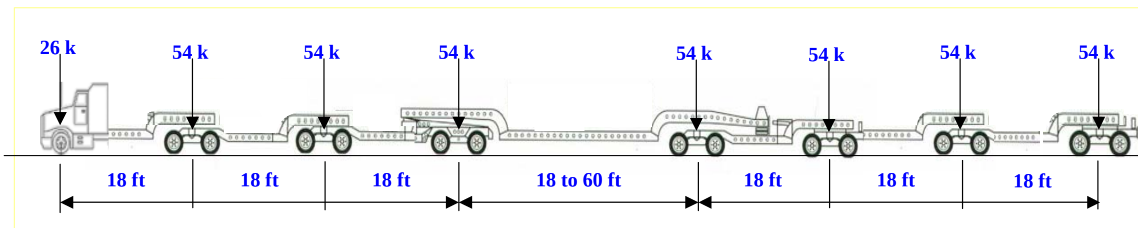

Figure 3.6.1.8.1-1 California P15 truck

3.6.1.8.2. Application

The permit design live loads shall be applied in combination with other loads as specified in Article 3.4.1. Axles that do not contribute to the extreme force effect under consideration shall be neglected.

Dynamic load allowance shall be applied as specified in 3.6.2.

Multiple presence factors shall be applied as specified in Article 3.6.1.1.2. However, when only one lane of permit is being considered, the MPF for one loaded lane shall be 1.0.

3.6.2 Dynamic Load Allowance: *IM*
3.6.2.1 General

Revise the 1st paragraph as follows:

Unless otherwise permitted in Articles 3.6.2.2 and 3.6.2.3, the static effects of the design truck, ~~or~~ design tandem, or permit vehicle other than centrifugal and braking forces….

Revise Table 3.6.2.1-1 as follows:

Component	*IM*
Deck Joints—All Limit States	75%
All Other Components	
• Fatigue and Fracture Limit State	15%
• Strength II Limit State	25%
• All Other Limit States	33%

C3.6.2.1

Revise the 4th and 5th paragraphs as follows:

Field tests indicate that in the majority of highway bridges, the dynamic component of the response does not exceed 25 percent of the static response to vehicles. This is the basis for dynamic load allowance with the exception of deck joints. However, the specified live load combination of the design truck and lane load, represents a group of exclusion vehicles that are at least 4/3 of those caused by the design truck alone on short- and medium-span bridges. The specified value of 33 percent in Table 1 is the product of 4/3 and the basic 25 percent. California removed the 4/3 factor for Strength II because a lane load isn't a part of the design permit vehicle used. Furthermore, force effects due to shorter permit vehicles approach those due to the HL93. The HL93 tandem*1.33 + lane generally has a greater force effect than that due to the P15 on short-span bridges.

Generally speaking, the dynamic amplification of trucks follows the following general trends:

- As the weight of the vehicle goes up, the apparent amplification goes down.
- Multiple vehicles produce a lower dynamic amplification than a single vehicle.
- More axles result in a lower dynamic amplification.

~~For heavy permit vehicles which have many axles compared to the design truck, a reduction in the dynamic load allowance may be warranted.~~ A study of dynamic effects presented in a report by the Calibration Task Group (*Nowak 1992*) contains details regarding the relationship between dynamic load allowance and vehicle configuration.

3.6.3 Centrifugal Forces: *CE*

Revise as follows:

For the purpose of computing the radial force or the overturning effect on wheel loads, the centrifugal effect on the live load shall be taken as the product of the axle weights of the design truck, ~~or~~ design tandem, or permit vehicle and the factor C, taken as:

(no change to equation)

Highway design speed shall not be taken to be less than the value specified in AASHTO publication *A policy of Geometric Design of highways and Streets (1990),* the Caltrans Highway Design Manual (current edition), or as otherwise directed. The design speed for permit vehicles shall be 25 mph, maximum.

The multiple presence factors specified in Article 3.6.1.1.2 shall apply.

Centrifugal forces ~~shall~~ may be applied horizontally at a distance 6.0 ft above the roadway surface. A load path to carry the radial force to the substructure shall be provided. The effect of superelevation in reducing the overturning effect of centrifugal force on vertical wheel loads may be considered.

C3.6.3

Revise the 4th paragraph as follows:

Centrifugal force ~~also~~ does cause~~s~~ an overturning effect on the wheel loads when ~~because~~ the radial force is applied 6.0 ft. above the top of the deck. Thus, centrifugal force tends to cause an increase in the vertical wheel loads toward the outside of the bridge and an unloading of the wheel loads toward the inside of the bridge. The effect is more significant on structures with single column bents, but can be ignored for most applications. Superelevation helps to balance the overturning effect due to the centrifugal force and this beneficial effect may be considered. The effects due to vehicle cases with centrifugal force effects included should be compared to the effects due to vehicle cases with no centrifugal force, and the worst case selected.

3.6.4 Braking Force: *BR*

Revise the 1st paragraph as follows:

The braking force shall be taken as the greater of:

- 25 percent of the axle weights of the design truck or design tandem or,
- percent of the design truck plus lane load or 5 percent of the design tandem plus lane load

This braking force shall be placed in all design lanes which are considered to be loaded in accordance with Article 3.6.1.1.1 and which are carrying traffic headed in the same direction. These forces shall be assumed to act horizontally at ~~a distance of 6.0 ft above~~ the roadway surface in either longitudinal direction to cause extreme force effects. All design lanes shall be simultaneously loaded for bridges likely to become one-directional in the future.

C3.6.4

Revise the 1st paragraph as follows:

Based on energy principles, and assuming uniform deceleration, the braking force determined as a fraction of vehicle weight is:

$$b = \frac{v^2}{2ga} \qquad \text{(C3.6.4-1)}$$

where a is the length of uniform deceleration and b is the fraction. Calculations using a braking length of 400 ft. and a speed of 55 mph yield b = 0.25 for a horizontal force that will act for a period of about 10 seconds. The factor b applies to all lanes in one direction because all vehicles may have reacted within this time frame. The overturning effect from braking is dependent on the number of axles and location of the drive train. This load may be applied at deck level with negligible effect on member sizes.

Delete Article 3.6.5.2 and Commentary

~~3.6.5.2 Vehicle and Railway Collision with Structures~~

~~Unless protected as specified in Article 3.6.5.1, abutments and piers located within a distance of 30.0 ft. to the edge of roadway, or within a distance of 50.0 ft. to the centerline of a railway track, shall be designed for an equivalent static force of 400 kip, which is assumed to act in any direction in a horizontal plane, at a distance of 4.0 ft. above ground.~~

~~The provisions of Article 2.3.2.2.1 shall apply.~~

~~C3.6.5.2~~

~~It is not the intent of this provision to encourage unprotected piers and abutments within the setbacks indicated, but rather to supply some guidance for structural design when it is deemed totally impractical to meet the requirements of Article 3.6.5.1.~~

~~The equivalent static force of 400 kip is based on the information from full-scale crash tests of barriers for redirecting 80.0-kip tractor trailers and from analysis of other truck collisions. The 400-kip train collision load is based on recent, physically unverified, analytical work (*Hirsch 1989*). For individual column shafts, the 400-kip load should be considered a point load. For wall piers, the load may be considered to be a point load or may be distributed over an area deemed suitable for the size of the structure and the anticipated impacting vehicle, but not greater than 5.0 ft. wide by 2.0 ft. high. These dimensions were determined by considering the size of a truck frame.~~

3.7.5 Change in Foundations Due to Limit State for Scour

Revise as follows:

The provisions of Article 2.6.4.4 shall apply. The effects due to 0% channel degradation and 100% channel degradation shall be considered. In addition, the effects due to 100% channel degradation plus 50% local contraction scour shall be considered in all strength limit state load combinations.

The consequences of changes in foundation conditions resulting from the ~~design and~~ Q_{100} base ~~and check~~ flood~~s~~ ~~for scour~~ shall be considered as specified in Section 2, and Articles 3.4.1 and 10.5 of the Specifications and California Amendments.~~at strength and service limit states. The consequences of changes in foundation conditions due to scour resulting from the check flood for bridge scour and from hurricanes shall be considered at the extreme event limit states.~~

C3.7.5

Revise as follows:

Statistically speaking, scour is the most common reason for the failure of highway bridges in the United States.

Provisions concerning the effects of scour are given in Section 2. Scour per se is not a force effect, but by changing the conditions of the substructure it may significantly alter the consequences of force effects acting on structures. The design for fully-factored live loads in the scour conditions described for the strength limit state is in lieu of designing for an extreme event for flood.

3.8.1.3 Wind Pressure on Vehicles: *WL*

Revise the 1st paragraph as follows:

When vehicles are present, the design wind pressure shall be applied to both structure and vehicles. Wind pressure on vehicles ~~shall~~ may be represented by a~~n interruptible,~~ moving force of 0.10 klf acting normal to, and 6.0 ft. above the roadway and shall be transmitted to the structure.

C3.8.1.3

Add a new last paragraph as follows:

Force effects due to this overturning couple of the vehicle are negligible in structures on piers and multi-column bents, and can be ignored for most applications. If the load is applied at deck level rather than 6.0 ft. above the deck, the effect on member sizes is negligible.

3.12 FORCE EFFECTS DUE TO SUPERIMPOSED DEFORMATIONS: *TU*, *TG*, ~~*SH*, *CR*~~, *SE*

Delete Article 3.12.2 and replace with the following:

3.12.2 Uniform Temperature

The design thermal movement associated with a uniform temperature change ~~may~~ shall be calculated using Procedure A. ~~or Procedure B below. Either Procedure A or Procedure B may be employed for concrete deck bridges having concrete or steel girders. Procedure A shall be employed for all other bridge types~~.

3.12.2.1 Temperature Range for Procedure A

The ranges of temperature shall be as specified in Table 1. The difference between the extended lower or upper boundary and the base construction temperature assumed in the design shall be used to calculate forces due to thermal deformation~~ effects~~. Force effects shall be calculated using gross section properties and the lower value for γ_{TU}.

Unless otherwise specified, ~~T~~the minimum and maximum temperatures specified in Table 1 shall be taken as $T_{minDesign}$ and $T_{maxDesign}$ respectively, in Eqs. 1 and 2.

The design thermal movement range for force effects, Δ_T, shall be investigated for both of the following:

$$\Delta_T = \alpha L(T_{maxDesign} - T_{BaseConstr}) \quad \text{(Eq. 3.12.2.1-1)}$$

$$\Delta_T = \alpha L(T_{minDesign} - T_{BaseConstr}) \quad \text{(Eq. 3.12.2.1-2)}$$

where:
$T_{BaseConstr}$ = base construction temperature (°F)

L = expansion Length (in.)

α = coefficient of thermal expansion (in./in./°F)

C3.12.2
Add as follows:

The designer should make appropriate allowances for avoiding the possibility of hard surface contact between major structural components. Such conditions include the contact between slotted holes and anchor bolts, and between girders and abutments. Expansion joints and bearings should account for differences between the setting temperature and an assumed design installation temperature.

3.12.2.2 Temperature Range for Procedure B
Delete contents of the entire Article including Commentary and Figures.

3.12.2.3 Design Thermal Movements

The design thermal movement range, Δ_T, for joints and bearings, shall depend upon the extreme bridge design temperatures defined in Article 3.12.2.1 ~~or 3.12.2.2~~ and be determined as:

$$\Delta_T = \alpha L(T_{maxDesign} - T_{MinDesign}) \quad \text{(Eq. 3.12.2.3-1)}$$

where:

L = expansion length (in.)

α = coefficient of thermal expansion (in./in./°F)

3.12.4 Differential Shrinkage

C3.12.4
Add a new last sentence as follows:
The load factor may be reduced to 1.0 if **physical** testing **approved by the Owner** is performed to establish material properties.

3.12.5 Creep

C3.12.5
Add a new last sentence as follows:
The load factor may be reduced to 1.0 if **physical** testing **approved by the Owner** is performed to establish material properties.

Add a new Article 3.12.7 and Commentary, as follows:

3.12.7 Secondary Forces from Post-Tensioning, *PS*
The application of post-tensioned prestress forces on a statically indeterminate structure, produces reactions at the structure's support and internal forces that are collectively called secondary forces.

C3.12.7

In frame analysis software, secondary forces are generally obtained by subtracting the primary prestress forces from the total prestresssing.

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